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Seismic Retrofit of Existing Concrete Frame Structures Using Viscoelastic Damping Devices

A Research-in-Progress Update

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Foreword

This study was conducted for Headquarters, U.S. Army Corps of Engineers (HQ-USACE) under Project 4A162784AT41, "Military Facilities Engineering Technology"; Work Unit FM-CW4, "Seismic Retrofit Techniques for Existing Concrete Buildings." The technical monitor was Charles Gutberlet, CEMP-ET.

The work was performed by the Engineering Division (FL-E) of the Facilities Technology Laboratory (FL), U.S. Army Construction Engineering Reserch Laboratories (USACERL). The USACERL principal investigators were John Hayes and Pamalee Brady. Elizabeth Dausman, Brian Umbright, Douglas Foutch, and Sharon Wood are associated with the Department of Civil Engineering, University of Illinois, Urbana-Champaign (UIUC). Appreciation is given to the various organizations that supported this research: the U.S. Army Corps of Engineers and the National Center for Earthquake Engineering Research for research funding; 3M Corporation for donated engineering expertise and damper devices; and Ivy Wire and Steel for donated steel reinforcement. Larry M. Windingland is Chief, CECER-FL-E, Donald F. Fornier, Jr. is Acting Operations Chief, and Alvin Smith is Acting Chief, CECER-FL. The USACERL technical editor was William J. Wolfe, Information Management Office.

LTC David J. Rehbein is Commander and Acting Director of USACERL, and Dr. Michael J. O'Connor is Technical Director.

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1 Introduction

The U.S. Army and Air Force have large inventories of concrete frame buildings that were constructed before the 1970s. Older concrete building structures were not designed using the stringent detailing requirements of today's building codes (e.g., the Uniform Building Code, or the American Concrete Institute [ACI] Building Code Requirements for Reinforced Concrete) because seismic provisions in most U.S. building codes were not enhanced until after the 1971 San Fernando earthquake.

Consequently, older buildings are generally not considered to be as capable as structures designed according to current codes of safely withstanding ground motions associated with large intensity earthquakes, i.e., lateral motions, without significant damage. Common shortcomings of older structures include the lack of sufficient stirrups around the primary reinforcement in columns, poor reinforcement lap splice location and length in columns, and lack of continuity of slab and/or girder reinforcement in joint regions. Furthermore, current design practice generally precludes placing a flat slab structure in a seismically active region. Problems associated with shear and moment transfer between slabs and columns lead to premature punching shear failures of slabs near columns when such structures are loaded laterally, as in earthquakes. Numerous older buildings on military installations are susceptible to earthquake damage because of these detailing problems.

In addition to the general structural safety problem associated with older structures, the military is faced with the more subtle problem of excessive earthquake-induced displacements in structures that house critical equipment (e.g., hospitals). While structural safety itself may not be a problem, large, violent building motions can damage critical and expensive equipment inside buildings, or lead to costly and dangerous damage to architectural features (e.g., suspended ceilings).

Traditional means of rehabilitating these buildings to resist earthquake-induced loads can be costly to implement and disruptive to functions housed in them. Researchers at the U.S. Army Construction Engineering Research Laboratories (USACERL) have developed a research program that focuses on providing improved technologies for mitigating seismic hazards in older military buildings. This ongoing, multi-year project examines the use of viscoelastic damping devices to provide passive energy dissipation in seismic rehabilitations of existing, lightly reinforced, concrete frame structures. Research to date has included both analytical modeling and experimental

testing of scaled model concrete structures on the USACERL shaking table. USACERL researchers have proposed broadening these research efforts to include analytical and experimental evaluations of other means of passive energy dissipation, to provide a complete data base of the various supplemental damping options that have entered the U.S. market in recent years.

Because the USACERL research involves the study of conventional buildings, it has attracted interest from outside the Department of Defense. The project represents a unique partnership of military, academic, and private sector organizations. The National Center for Earthquake Engineering Research (NCEER), Buffalo, NY, which has been studying the use of viscoelastic damping devices for both concrete and steel structures, joined the USACERL project and has supported the work of researchers at the University of Illinois at Urbana-Champaign (UIUC). Additionally, the Vibration Damping Program Office, 3M Corporation, St. Paul, MN, contributed to the NCEER project by providing damping devices used in the experimental phases of the project and by consulting with project researchers on the dampers' performance characteristics.

USACERL and UIUC jointly conducted a number of beam-slab-column and flat slab-column joint subassemblage tests on the USACERL shaking table in 1992 and 1993. Analysis of the data from those tests is underway, and reports on the results are forthcoming. In early 1994, USACERL and UIUC tested a one-third scale model of a three-story flat slab-column structure on the USACERL shaking table. This update presents a brief overview of the 1994 testing phase of the project.

Objectives

The overall objective of this research is to develop inexpensive, easy-to-implement, nonintrusive seismic rehabilitation techniques for existing buildings. The objective of this preliminary investigation was to examine the use of viscoelastic damping devices to provide passive energy dissipation in seismic rehabilitations of existing, lightly reinforced, concrete-frame structures, including analytical modeling and experimental testing of scaled model concrete structures on the USACERL shaking table.

Approach

1. A preliminary investigation identified the installation of viscoelastic damping devices as a possible technique for reinforcing concrete structures against seismic motion.
2. A prototype reinforced, concrete-frame structure was selected for its typical characteristics in terms of construction period and design.
3. Several one-half scale models of the beam-slab-column joint regions of the prototype structure were built and tested on the USACERL shaking table.
4. A one-third scale model of the prototype structure was built and placed on the USACERL shaking table.
5. Earthquake testing was done using two different sets of viscoelastic dampers, and with no dampers in place.
6. Data taken during each of the simulations was analyzed, preliminary observations were made, and the basis for further analysis and testing was formed.

Scope

It should be stressed that this report presents only an overview of, and preliminary results drawn from the 1994 testing phase of the project. More detailed analyses of the test data are underway. Design issues must be resolved before implementation of the described technology in a full-scale seismic rehabilitation program. For example, some issues still to be addressed include damper sizing and placement, ambient temperature control, damper proof testing, and appropriate structural detailing.

Points of Contact

Further information on these tests or related USACERL research is available through John Hayes at 217/373-7248, or Pamalee Brady at 217/373-7247.

Mode of Technology Transfer

The results of this preliminary investigation will form the basis of a more thorough examination of the data derived from this experiment, and to formulate further testing of this and other damping devices.

2 Experiment Overview

Description of the Technology

The rehabilitation technique studied in this project involves adding diagonal braces to the concrete structure. Each brace is fabricated from two structural sections that are linked with a viscoelastic damping device. The diagonal braces extend from the bottom of one column to the top of an adjacent column (Figure 1). The steel sections of the brace are attached to the reinforced concrete columns using steel collars that are bolted to and around the columns.

Figure 2 shows a typical viscoelastic damper. Four layers of viscoelastic material are glued (using epoxy cement) to steel plates. The dampers are bolted into the diagonal braces. The tensile and compressive strut forces that develop in the braces when the structure is subjected to earthquake loading are carried in direct shear by the layers of viscoelastic materials. If the dampers were not in the braces (i.e., if the braces were continuous steel members), the braces would stiffen the structure. This change in structural stiffness would change the displacement and acceleration responses of the structure to a given earthquake motion.

With the addition of the viscoelastic dampers in the braces, not only is the rehabilitated structure stiffer than the original, but the energy dissipation capacity of the structure is enhanced without inducing more structural damage. As the viscoelastic layers in the damping devices are sheared by the extensions and compressions of the braces, the material heats, as the energy being input to the structure by the ground motion is converted into heat energy. This energy dissipation in effect increases the total damping of the structure. Increased damping reduces both displacement and acceleration responses to a given ground motion.

The general effect of the stiffening and added damping provided by the dampers may be seen by examining displacement and acceleration response spectra for a single-degree-of-freedom (SDOF) oscillator for the 1940 El Centro earthquake record (Figures 3 and 4). The time scale for the El Centro motion in these figures has been compressed by a factor of $1/\sqrt{3}$, which was used in the experimental tests.

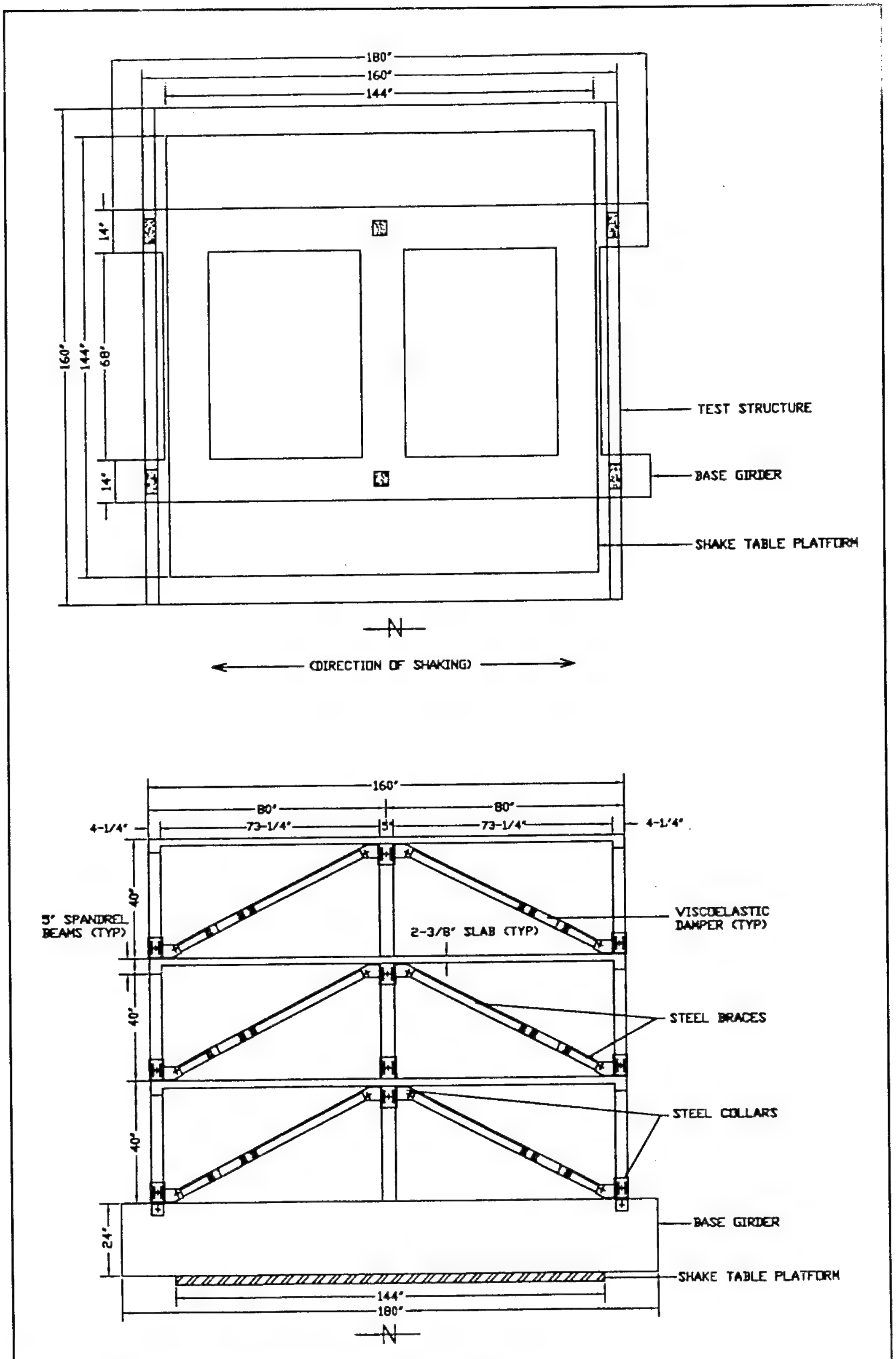

Figure 1. Placement of diagonal braces between columns.

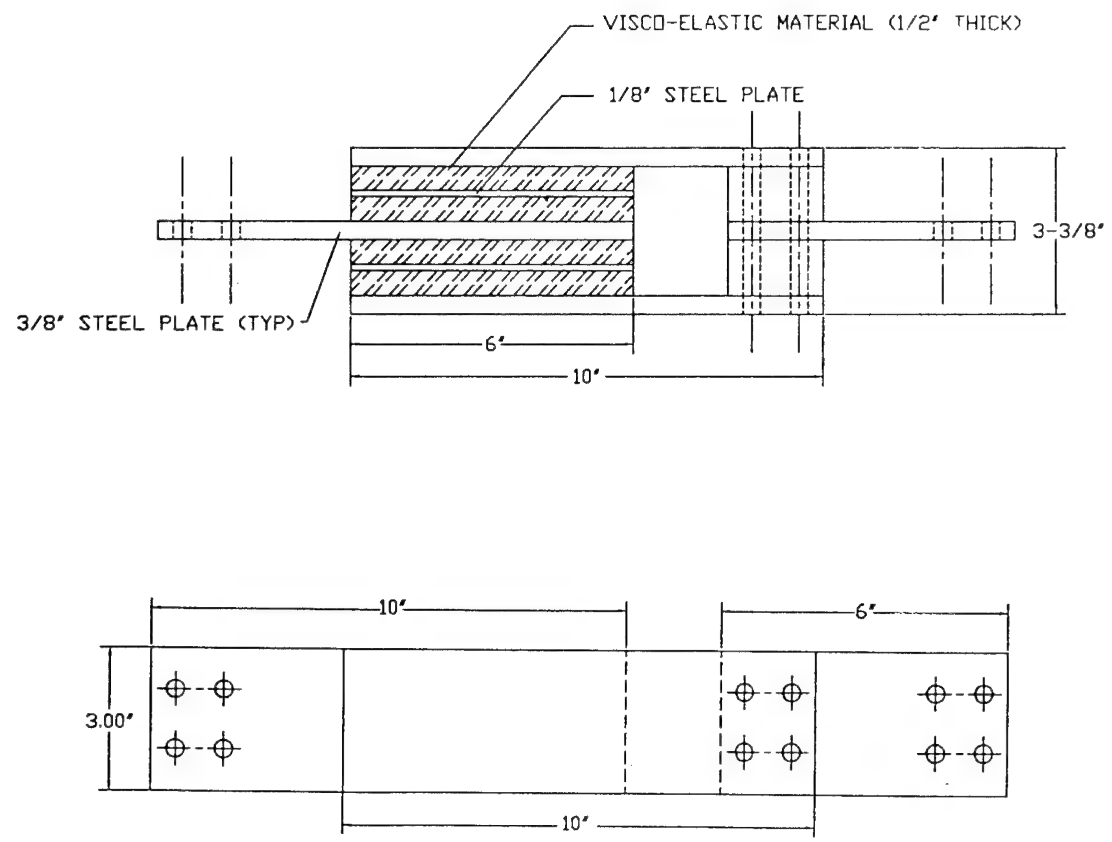

Figure 2. Typical viscoelastic damper.

An undamaged concrete structure without supplemental damping may be expected to exhibit an equivalent viscous damping factor of approximately 0.07 under service loads. The results of previous research indicate that the same structure with viscoelastic dampers installed may be reasonably expected to exhibit an equivalent viscous damping factor of 0.25. The reinforced concrete model tested in this program had a measured fundamental period of approximately 0.55 seconds without dampers. When the braces and dampers were added, this was reduced to approximately 0.30 seconds.

The acceleration response spectra show that increasing the stiffness of an SDOF oscillator in this period range without changing the amount of damping leads to large acceleration response increases. This is illustrated by moving from point A to point B on the acceleration response spectrum in Figure 4.

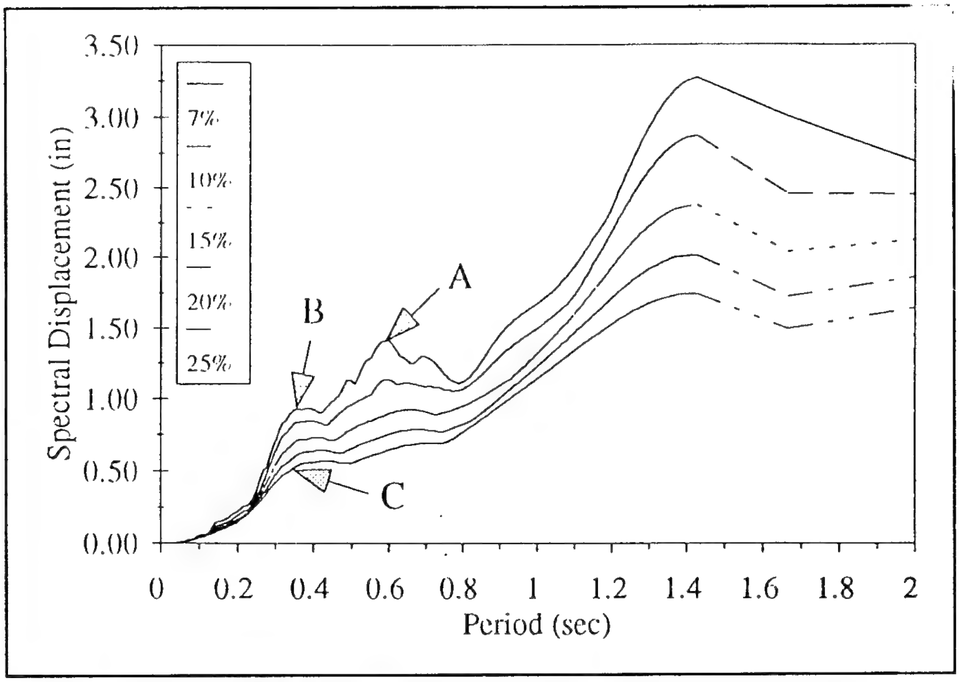

Figure 3. Spectral acceleration response for an SDOF oscillator for the El Centro earthquake (compressed time scale).

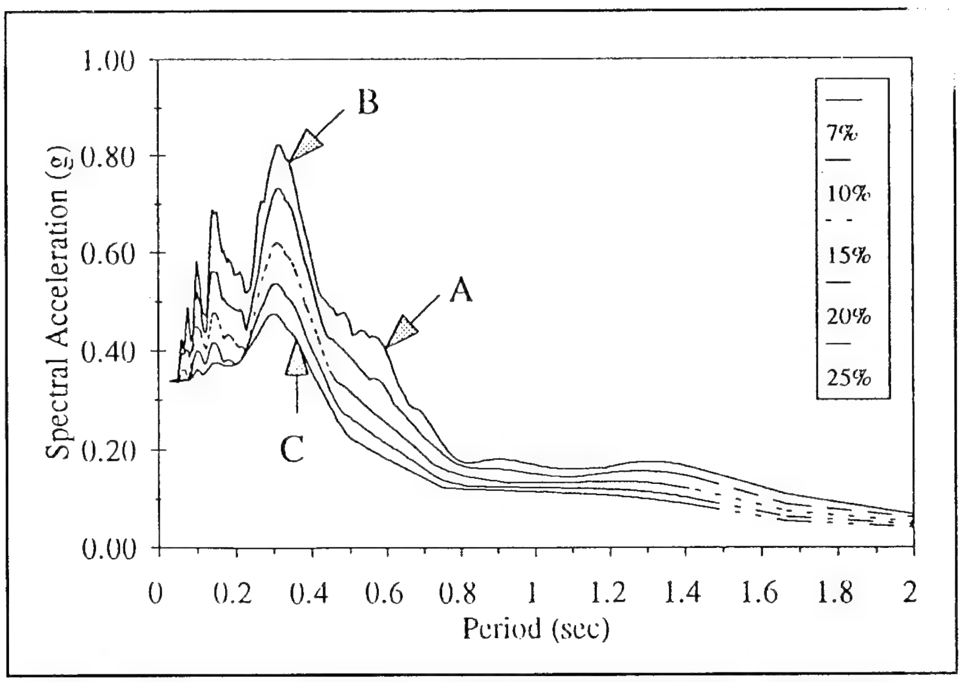

Figure 4. Spectral displacement response for an SDOF oscillator for the El Centro earthquake (compressed time scale).

Since the forces in the SDOF oscillator can be obtained by multiplying the structural mass by the acceleration, forces are seen to increase. With the added damping, which may be illustrated by moving to point C on the acceleration response spectra, peak acceleration, hence peak force, is reduced substantially from that obtained by

increasing stiffness without adding damping. More significantly, the displacement response spectra show that peak displacements are reduced considerably more when the added damping is considered than when stiffening alone is considered. Stiffening without added damping can be shown by moving from point A to point B on the displacement response spectra (Figure 3). Added damping could result in moving to point C in Figure 3.

The viscoelastic material has two key properties that must be considered in the design process. First, the effective shear stiffness of the material depends on the frequency content of its motion. As the frequency of shearing excitations increases, so does the effective stiffness. Second, the effective stiffness of the material depends on its temperature. As its temperature increases, its stiffness decreases, and vice versa. This latter property is particularly critical to building design, as a damper using the material must be designed to operate in a specified ambient temperature range; making it therefore likely that viscoelastic dampers will require placement in a controlled temperature environment. The target temperature for the testing in this program was 72 °F.*

Prototype Structure

The prototype building for this model is a barracks building at Fort Lewis, WA. The building is actually an "H" shaped complex of three structures, one wing of which served as the prototype for these tests. The prototype wing is rectangular in plan, with a width of approximately 40 ft and a length of approximately 117 ft. It is three stories high. The structural framing system is predominately a three-story, reinforced concrete, column-flat slab system. Cast-in-place shear walls at the ends of the long dimension provide lateral force resistance for transverse ground motions, but there are no intermediate shear walls in that direction. In the longitudinal dimension, spandrel beams run the length of the building on both exterior walls, at the top of each story. The spandrels, which are cast monolithically with the floor slabs, support exterior wall and window systems and stiffen the framing system. Columns are founded on individual spread footings.

As-built drawings indicate the building was constructed circa 1956. Design calculations were not available, but the drawings indicate the designers used the Pacific Coast Uniform Building Code (PCUBC) and the ACI Building Code (ACIBC). All analyses for the project are based on the assumption that the 1955 PCUBC and 1951 ACIBC were used, because those codes were in use at the time of construction.

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* °F = (°C × 1.8) + 32; 1 ft = 0.305 m; 1 psi = 703.1 kg/m^2.

The drawings further indicate that the structure was designed to PCUBC seismic Zone III provisions. Structural concrete was designed for a 28-day compressive strength of 3,000 psi; reinforcing steel was designed for a working stress of 20,000 psi.

Review of the structural drawings pointed out a number of obvious potential problem areas in the structural design, compared with current building code provisions. First, the structure is a flat slab structure. The slab bottom (positive moment) reinforcement is not continuous through interior column regions. Column ties are smaller and spaced further apart than required in today's codes. No column reinforcement lap splice details are shown on the drawings, although splice lengths are given. The splice lengths are shorter than would be required today. Because splice locations are not shown on the structural drawings, splices were assumed to be located in accordance with common practice of that period, directly above each floor level. Stirrups were also widely spaced in the spandrel beams, limiting both their flexural steel confinement and their torsional capacity. Each of these factors acts to limit the structure's ability to displace laterally under the effects of earthquake-induced ground motions.

Model Structure

A transverse section near the center of the prototype structure was selected for the shaking table tests. This region was considered the most vulnerable area in the structure, as it was a large distance from the perimeter shear walls. The model was constructed at one-third of full scale. Figure 5 shows the overall dimensions and details of the test structure. Reinforcement details in the model were essentially the same as those in the prototype.

The test structure was formed and cast in four levels. The base girder was cast first, followed by the floor and supporting columns in each successively higher story. Ready-mixed concrete with a specified compressive strength of 3,500 psi was used. This strength was chosen to simulate the strength increase with age that would be expected in the prototype. USACERL staff technicians constructed the formwork, while other USACERL staff technicians and UIUC students placed the concrete.

Insofar as possible, the size of the reinforcement was also scaled to the one-third of the size in the prototype. Deformed steel wire from welded wire fabric, donated to the project by Ivy Wire and Steel, Houston, TX, was used to model individual reinforcement bar diameters as closely as possible. The prototype reinforcement was assumed to have a nominal yield stress of 40,000 psi, since the as-built drawings specified a working stress of 20,000 psi.

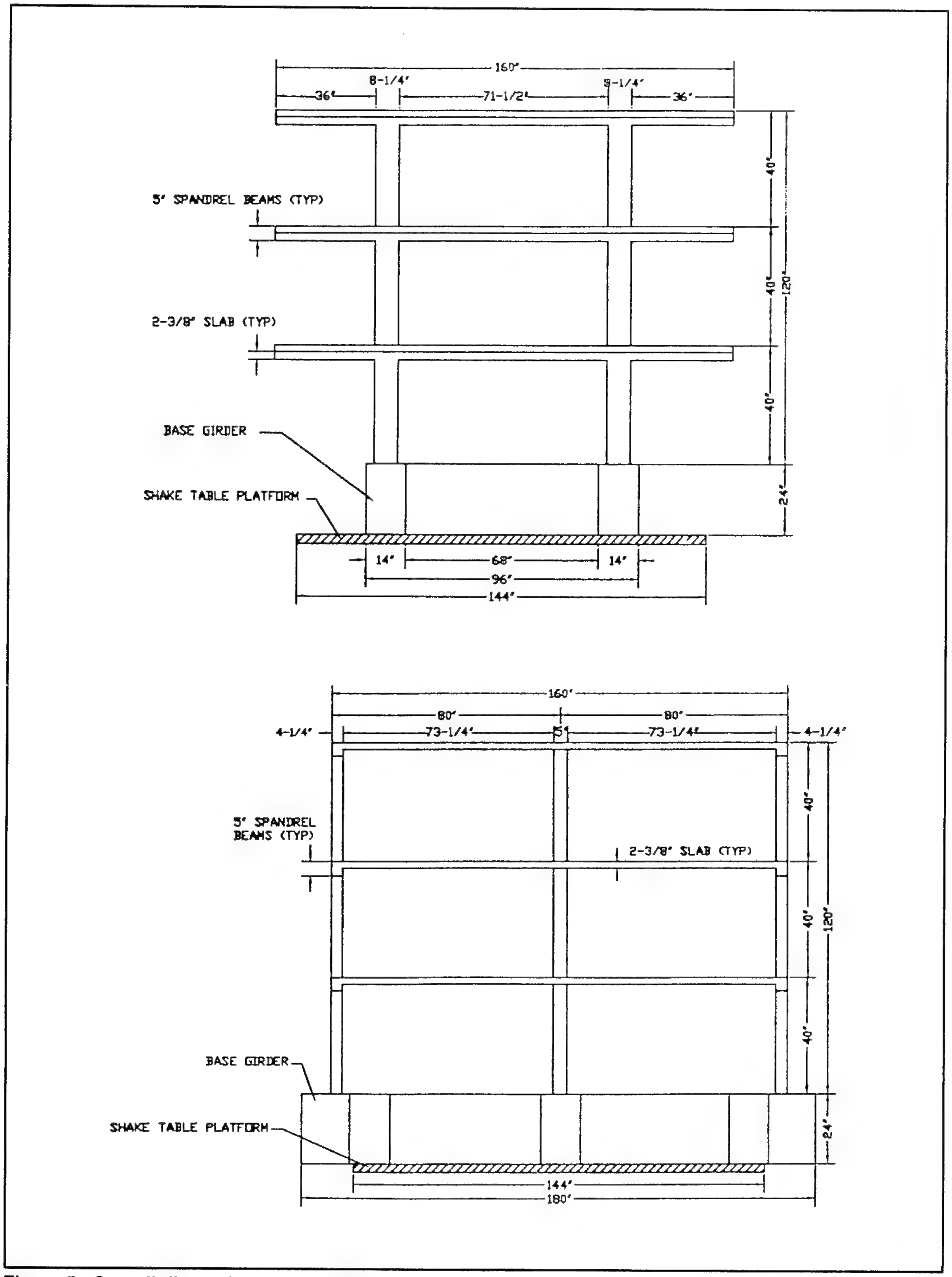

Figure 5. Overall dimensions and details of test structure.

The wire used in the model initially had a yield stress approaching 100,000 psi, so all wire used for longitudinal reinforcement in the slabs, columns, and spandrels was annealed as needed to match the prototype strength. Column and spandrel transverse

reinforcement was not annealed to save the time and expense of the annealing. The wide tie spacing and the consequent lack of primary steel confinement in the prototype were believed to more critical than tie strength.

The base girder was cast as a large, stiff, monolithic unit to accommodate construction of the model structure off the shaking table platform. The girder was then used to lift the model onto the table for the earthquake simulations. Rather than attaching overhead crane pickup points to the model, the crane pickups were attached directly to the base girder, which fully supported the model. The first-story columns were cast over longitudinal reinforcement that was stubbed out of the base girder. The presence of the base girder increased the base fixity of the first-story columns in the model beyond that which might be expected in the prototype, where individual column spread footings would permit some base rotation to occur.

To simulate the gravity load stresses in the various structural elements, and to maintain proper dynamic response characteristics in the structure, it was necessary to add lead ingots as floor masses in the model structure. The average total floor load, including the ingots, in the model approximately 100 psf,* which was very close to the total floor load in the prototype structure.

"Rehabilitation" of the Model Structure

The overall objective of this research is to develop inexpensive, easy-to-implement, and nonintrusive seismic rehabilitation techniques for existing buildings. Therefore, the test model was constructed to represent the as-built condition of the prototype structure as closely as possible, and then a rehabilitation scheme was designed for the model. As described earlier, damping devices were added in the column lines of the test structure by adding diagonal braces that contained the dampers. The braces were attached to the columns by means of steel collars that were bolted in place on the columns. Researchers measured the column dimensions and fabricated the columns to provide a snug fit when they were bolted in place. Before the collars were installed, a thin sand-cement mortar grout was troweled on the exposed column dimensions. Researchers then bolted the collars in place while the grout was still wet. In this manner, the collars not only served to transfer the forces that were transmitted by the damper braces, they also provided confinement of the column concrete, thereby increasing the shear and rotational capacities of the columns. In addition, to ensure that the collars did not slip along the column height, an all-thread bolt was placed through each column in the collar region and both sides of the collar. Because of

* 1 psf = 4.882 kg/m^2.

researchers' concern for damaging the scaled model column, the bolt was cast into the column during construction. In a full scale structure, such an attachment would be drilled and grouted in. Figure 1 shows how the damping devices and braces were placed in the test structure; Figure 6 shows a typical detail of an interior column-slab connection.

The results of structural analyses conducted before the experimental tests indicated that the damper braces would significantly increase shear forces at the bases of the first and second floor outside columns. To minimize the likelihood of a column shear failure, shear transfer capacity was increased at both levels. At the base girder level, the collars were connected to the girder, which represented a footing, by means of drilling into the base girder on each side of the column and epoxying in an anchor bolt that was in turn attached to the collar. On the first-story level, instead of drilling into the base girder, the researchers drilled and epoxied into the spandrel beam in a similar manner.

The only other significant strengthening measure that was taken was to increase slightly (approximately 20 to 30 percent) the overall column flexural strength at the connection to the base girder by using No. 3 bars of 60,000 psi steel, instead of the scaled wire reinforcement with a yield stress of 40,000 psi. Increasing the flexural capacities of these critical columns was assumed to be a likely, straightforward part of any rehabilitation scheme.

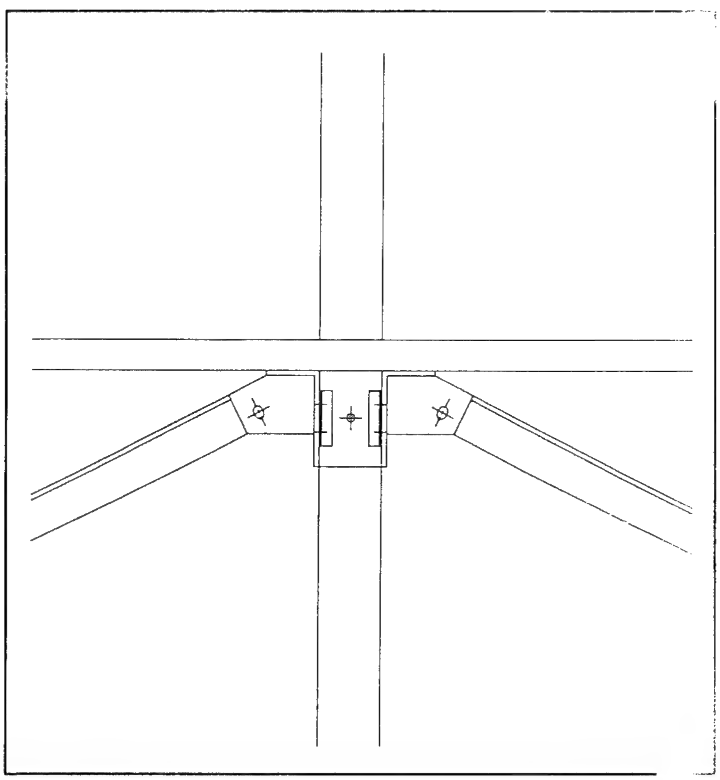

Figure 6. Detail of an interior column-slab connection.

3 Test Program

Following the completion of model construction and appropriate concrete curing time, the model was placed on the USACERL shaking table in early 1994. Researchers added lead ingots, constructed a separate steel emergency support structure (for use in the event of a catastrophic collapse), and installed test instrumentation. Approximately 75 channels of data were recorded in each test. Instrumentation included longitudinal and lateral accelerations and displacements of the shaking table, base girder, and each floor; damper displacements; damper brace force; damper temperature change (in selected dampers); and reinforcement strains in key column and slab locations.

Earthquake testing was conducted using two different sets of viscoelastic dampers from 3M, as well as with no dampers in place. 3M engineers fabricated one set of dampers for the desired degree of added damping. They fabricated the second set of dampers with half the volume of damping material of the first set. Prior to the earthquake testing, representative samples of the two sizes of dampers that had been fabricated were tested in static and cyclic tests, to verify theoretical damper characteristics and ensure sound fabrication. All earthquake testing was conducted with the steel collars in place, so that damper effects could be isolated and analyzed.

Before any earthquake simulations were conducted, researchers determined the modal (first three modes) frequencies and equivalent viscous damping of the undamaged structure in four different states: with no dampers or braces installed, with solid steel braces installed, with the small dampers installed in braces, and with the large dampers installed in braces. Two methods of determining the dynamic characteristics were used. First, a simple “pullback” test was used, in which a weight and pulley system was used to pull the structure laterally through an attachment at the top floor level; the weight was suddenly released. After the release of the weight, structural motions were recorded, from which modal responses and damping were analyzed. This method relies on the structure’s displacing in a manner that is dominated by its first mode. Second, the shaking table was used to input white noise into the model. By examining acceleration transfer functions between floors of the model, it was possible to determine the first three modes of response. After the white noise test, the table would be used to excite the structure with a sine wave input at its already-determined first mode response frequency. Researchers would then shut the table off and measure

the decay of the induced structural motion, from which equivalent viscous damping could be determined, using logarithmic decrement calculations.

After some trials with the pullback tests, researchers abandoned them. The shaking table tests were more precise and better-controlled. In addition, when dampers were installed, the damping ratio was so high that the structure would not oscillate during the pullback tests. It is also important to note that measuring the dynamic characteristics became far more difficult during the shaking table tests. The high degree of damping in the structure greatly broadened the transfer function bandwidths; peak response frequencies were quite difficult to identify.

For the earthquake tests with dampers installed in the model, which were conducted before any earthquake tests without dampers were conducted, two characteristic earthquake records were used. The first was the El Centro site record from the 18 May 1940, Imperial Valley, CA, earthquake. The second was the Taft site record from the 21 July 1952, Kern County, CA, earthquake. The two earthquake records differ significantly, in terms of peak amplitudes and frequency contents. Figures 7 and 8 show the two records with their time scales compressed to $1/\sqrt{3}$ of full scale. (This time scale was used for all earthquake simulations.) To equate the energy contents of the two records for the earthquake simulations, the researchers used the ratio of the spectrum intensities (Housner, 1959) of the two time-scaled records. This process resulted in a need to multiply the Taft motion amplitudes by a factor of 2.1 to incorporate the same energy content as the El Centro record.

Researchers conducted a series of low level earthquake simulations with the small dampers in place. The simulations were run with base acceleration amplitudes of 1, 10, and 25 percent of the El Centro acceleration record; and at 2.1, 21, and 52.5 percent of the Taft acceleration record. Then the structure was tested for the same earthquake records with large dampers installed. (One "mixed damper" test was also run with large dampers on the first floor and small dampers on the upper two floors.) After each earthquake simulation, the dynamic characteristics were determined using the previously described white noise and sine decay tests, and the model was checked visually for cracking or other deterioration. The measured dynamic characteristics indicated how the structure was deteriorating under testing; as it softened due to cracking of the concrete, the fundamental frequency decreased.

At the conclusion of the low-level tests, little, if any, damage had occurred in the model. Higher level earthquake simulations were then conducted using the large dampers. The larger dampers were the ones of original design configuration; their anticipated stiffness and damping characteristics were tuned to the structure.

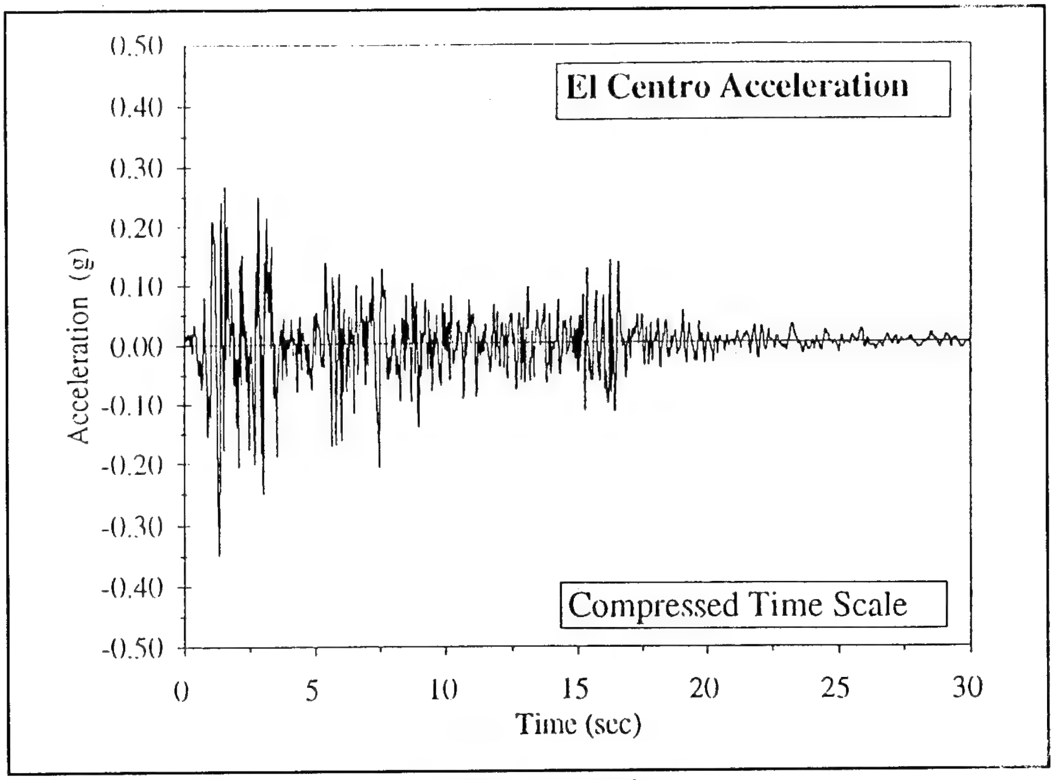

Figure 7. Time-scaled record from the El Centro earthquake.

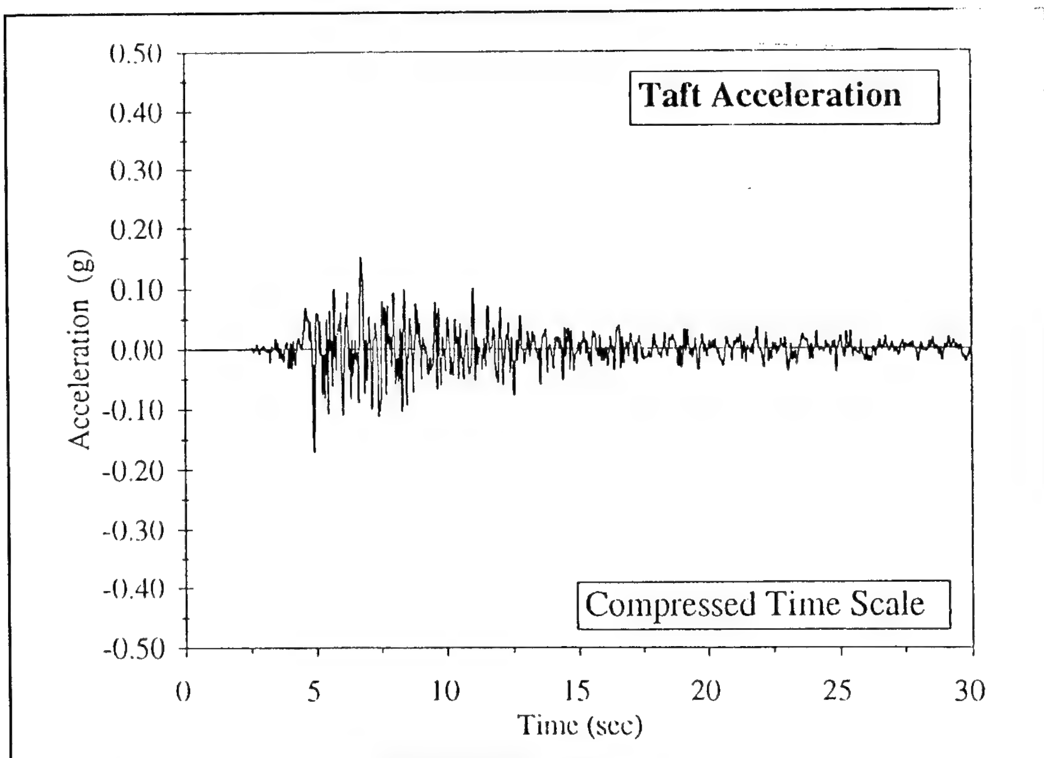

Figure 8. Time-scaled record from the Taft earthquake.

The higher level earthquake simulations were run at acceleration amplitudes of 10, 25, 50, 75, 100, 125, 150, 200, and 250 percent of the El Centro acceleration record; and at acceleration amplitudes of 21, 52.5, 105, 157.5, 210, 262.5, 315, and 420 percent of the Taft acceleration record. Because the higher level El Centro simulations included ground displacements exceeding the nominal ± 2-3/8-in. horizontal motion capacity of the shaking table, the 200-percent record was run with a 0.2 Hz high pass filter, while the 250-percent record was run with a 0.5 Hz high pass filter. This filtering of the earthquake record removed the lower frequency, larger displacements in the table motion.

After completing these earthquake simulations, the testing with dampers was terminated. The tests had reached both the displacement limits of the shaking table and the as-designed shear displacement limits of the dampers.

Following the completion of the tests with dampers installed, researchers removed the dampers from the braces in the structure. Subsequent tests were then run without dampers; the collars that were used to attach the dampers to the columns were left in place. Earthquake simulations were then run on the undamped, unbraced structure. The simulations included the same 25, 50, 75, 100, 125, 150, and 200-percent El Centro records that were used in the damper tests. Tests were concluded after the 200-percent El Centro simulation, when structural failure occurred. No Taft earthquake simulations of the undamped and unbraced structure were included in the test program.

4 Preliminary Observations and Future Work

While researchers have not yet performed detailed analysis of the experimental data, a few basic observations can be made.

When the testing began, a spurious oscillatory motion occurred in one of the six horizontal actuators that drive the shaking table, inducing torsion into the model briefly. Minor cracking of the floor slabs and spandrel beams occurred during the malfunction. At the time of the malfunction, no data acquisition channels were activated. However, a thorough check of all components of the structure revealed that no serious damage had occurred.

Initial white noise tests of the structure without braces or dampers showed a fundamental frequency of approximately 1.8 Hz. Based on logarithmic decrement measurements of the acceleration response decay after table shutoff, the baseline structure had an equivalent viscous damping factor of approximately 0.07 of critical damping. With the large dampers in place, the first mode frequency ranged between 3.05 and 3.56 Hz, for an average of about 3.3 Hz. The equivalent viscous damping based on the logarithmic decrement approached 20 percent of critical damping.

Preliminary indications are that the model with dampers sustained only minimal damage through the 150-percent El Centro earthquake simulation (peak ground acceleration (PGA) $\approx$ 0.55 g). The white noise tests following this earthquake test showed a fundamental frequency that was virtually unchanged from the initial white noise tests. Visual checks of the model showed only minor additional hairline cracking.

During the 150-percent El Centro earthquake test, the largest interstory drift was measured as 0.45 in. (1.1 percent), and it occurred in the second story. The total drift was 1.17 in. (1.0 percent). The maximum base shear experienced during the test was 34.6 kips; the total model weight, neglecting the base girder, was approximately 54.7 kips. Figures 9 and 10 show third floor displacement and base shear response histories, respectively. Tables 1 through 4 show representative tables of data from the test.

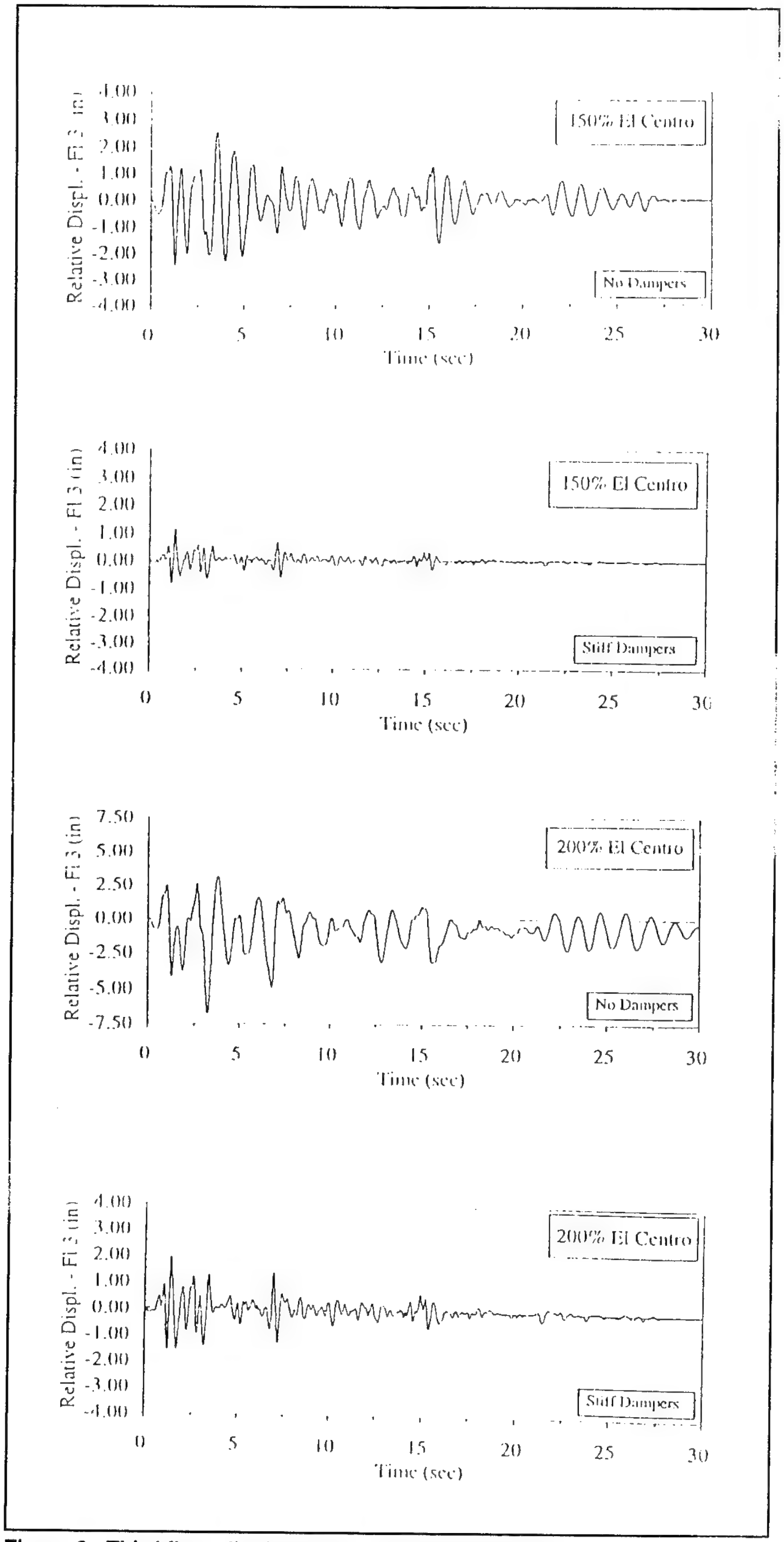

Figure 9. Third floor displacement response histories.

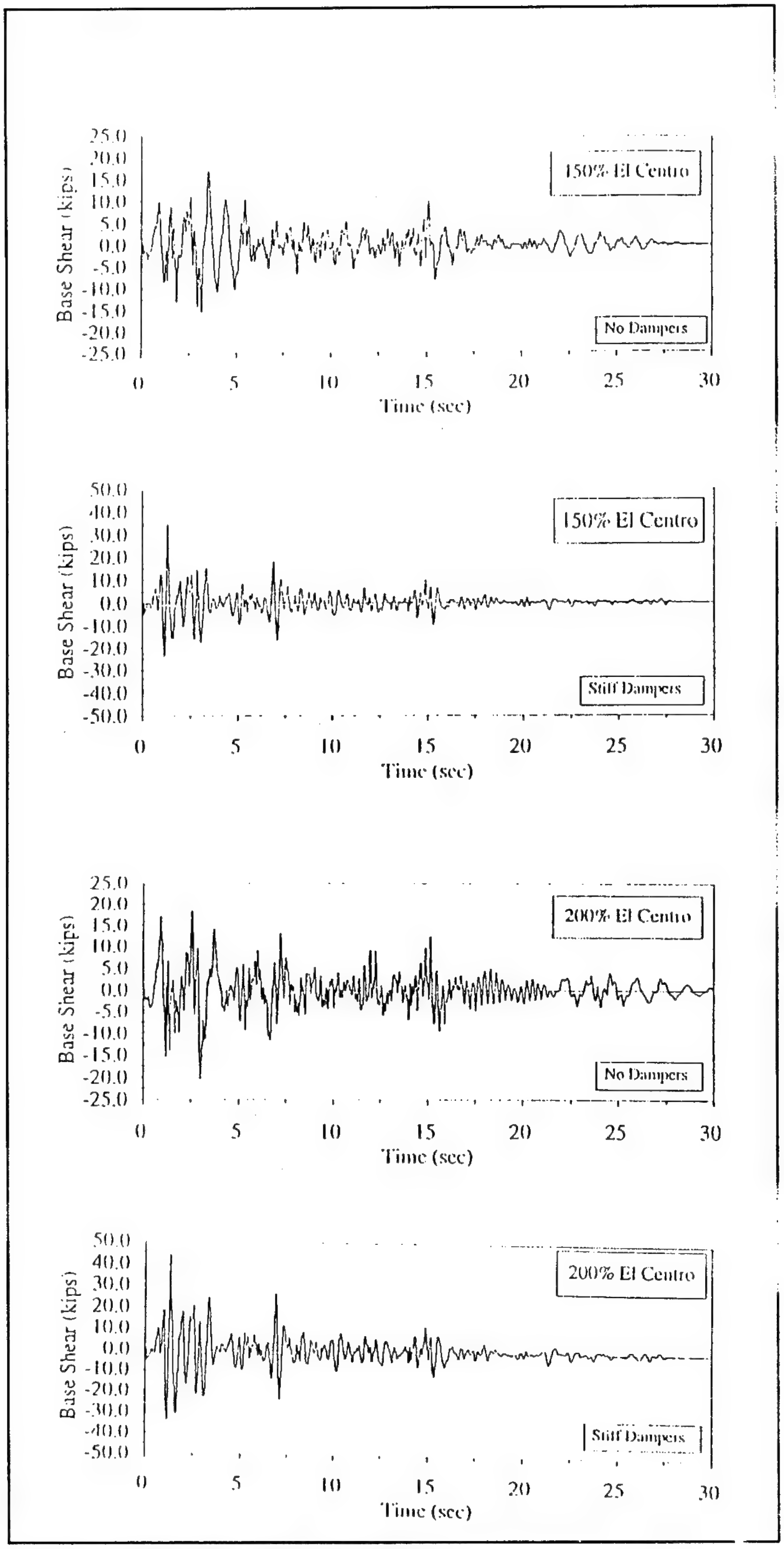

Figure 10. Base shear response histories.

It is important to note that corresponding El Centro and Taft earthquake tests were run sequentially, so, prior to the 150-percent El Centro test, the model had sustained the El Centro tests through the 100-percent level and the Taft tests through the 210-percent level. Maximum results of the Taft tests performed with the stiff dampers in place can be found in Tables 5 through 8.

The preliminary data review indicates that the model with dampers began to sustain discernable damage in the 200-percent El Centro earthquake test (PGA ≈ 0.86 g). The white noise tests following this earthquake test showed a fundamental mode frequency of approximately 2.88 Hz, showing some reduction in stiffness. Visual checks of the model showed additional hairline cracking had occurred. Most of this cracking was confined to flexural cracking in the floor slabs and minor torsional cracking of the spandrel beams at the interfaces with the columns. During the 200-percent El Centro earthquake test, the largest interstory drift was measured as 0.88 in. (2.2 percent), again occurring in the second story. The total drift was 2.04 in. (1.7 percent). The maximum base shear experienced during the test was 44.2 kips. Again, Figures 9 and 10 show third floor displacement and base shear response histories, respectively. Tables 1 through 4 show representative tables of data from the test.

Following the conclusion of the earthquake tests of the structure with dampers, the dampers were removed from the model, although the collars were left intact. For the earthquake tests without dampers, only the El Centro record was used. An initial white noise test showed a first mode frequency of approximately 1.56 Hz, versus the original 1.8 Hz. The model had softened and its measured damping had increased, indicating damage to the structure had accrued during the preceding earthquake simulation tests.

Table 1. Maximum absolute acceleration (g) with and without stiff dampers–El Centro.

Earthquake Test	Base Girder	1st Story	2nd Story	3rd Story
50% El Centro (w/dampers)	0.228	0.224	0.261	0.306
50% El Centro (no dampers)	0.212	0.213	0.229	0.178
100% El Centro (w/dampers)	0.408	0.397	0.475	0.598
100% El Centro (no dampers)	0.407	0.424	0.493	0.402
150% El Centro (w/dampers)	0.553	0.567	0.690	0.866
150% El Centro (no dampers)	0.563	0.640	0.602	0.472
200% El Centro (w/dampers)	0.858	0.779	0.959	1.247
200% El Centro (no dampers)	0.983	0.862	0.798	0.675

Table 2. Maximum relative displacement (in.) with and without stiff dampers–El Centro.

Earthquake Test	1st Story	2nd Story	3rd Story
50% El Centro (w/dampers)	0.124	0.255	0.346
50% El Centro (no dampers)	0.226	0.564	0.816
100% El Centro (w/dampers)	0.252	0.555	0.761
100% El Centro (no dampers)	0.943	1.591	2.033
150% El Centro (w/dampers)	0.412	0.847	1.134
150% El Centro (no dampers)	0.687	1.838	2.566
200% El Centro (w/dampers)	0.640	1.472	1.970
200% El Centro (no dampers)	0.862	3.897	6.814

Table 3. Maximum story shear (kips) with and without stiff dampers–El Centro.

Earthquake Test	1st Story	2nd Story	3rd Story
50% El Centro (w/dampers)	13.225	9.717	5.293
50% El Centro (no dampers)	5.180	4.263	3.092
100% El Centro (w/dampers)	23.707	18.553	10.360
100% El Centro (no dampers)	11.033	9.356	6.968
150% El Centro (w/dampers)	34.636	26.774	15.007
150% El Centro (no dampers)	16.987	14.414	9.811
200% El Centro (w/dampers)	44.219	37.108	21.592
200% El Centro (no dampers)	20.709	17.566	11.694

Table 4. Maximum inter-story displacement (in.) with and without stiff dampers–El Centro.

Earthquake Test	1st Story	2nd Story	3rd Story
50% El Centro (w/dampers)	0.124	0.139	0.096
50% El Centro (no dampers)	0.226	0.353	0.257
100% El Centro (w/dampers)	0.252	0.308	0.219
100% El Centro (no dampers)	0.943	0.710	0.541
150% El Centro (w/dampers)	0.412	0.453	0.306
150% El Centro (no dampers)	0.687	1.209	0.949
200% El Centro (w/dampers)	0.639	0.883	0.522
200% El Centro (no dampers)	0.864	3.076	2.967

Table 5. Maximum absolute acceleration (g) with stiff dampers–Taft.

Earthquake Test	Base Girder	1st Story	2nd Story	3rd Story
21% Taft (w/dampers)	0.046	0.046	0.036	0.051
52.5% Taft (w/dampers)	0.099	0.096	0.066	0.128
100% Taft (w/dampers)	0.175	0.166	0.203	0.233
157.5% Taft (w/dampers)	0.309	0.253	0.305	0.357
210% Taft (w/dampers)	0.414	0.348	0.425	0.489
262.5% Taft (w/dampers)	0.522	0.436	0.524	0.643
315% Taft (w/dampers)	0.628	0.530	0.562	0.722
420% Taft (w/dampers)	0.888	0.695	0.770	1.072

Table 6. Maximum story shear (kips) with stiff dampers–Taft.

Earthquake Test	1st Story	2nd Story	3rd Story
21% Taft (w/dampers)	2.199	1.494	0.886
52.5% Taft (w/dampers)	4.869	3.388	2.217
100% Taft (w/dampers)	9.544	6.465	2.871
157.5% Taft (w/dampers)	15.237	11.671	6.186
210% Taft (w/dampers)	19.693	16.368	8.463
262.5% Taft (w/dampers)	24.764	20.698	11.135
315% Taft (w/dampers)	28.583	22.451	12.509
420% Taft (w/dampers)	38.715	31.795	18.568

Table 7. Maximum relative displacement (in.) with stiff dampers–Taft.

Earthquake Test	1st Story	2nd Story	3rd Story
21% Taft (w/dampers)	0.025	0.051	0.076
52.5% Taft (w/dampers)	0.057	0.116	0.164
100% Taft (w/dampers)	0.120	0.241	0.320
157.5% Taft (w/dampers)	0.199	0.397	0.525
210% Taft (w/dampers)	0.268	0.575	0.761
262.5% Taft (w/dampers)	0.359	0.757	1.002
315% Taft (w/dampers)	0.425	0.891	1.172
420% Taft (w/dampers)	0.662	1.414	1.844

Table 8. Maximum inter-story displacement (in.) with stiff dampers–Taft.

Earthquake Test	1st Story	2nd Story	3rd Story
21% Taft (w/dampers)	0.025	0.032	0.028
52.5% Taft (w/dampers)	0.057	0.069	0.057
100% Taft (w/dampers)	0.120	0.134	0.083
157.5% Taft (w/dampers)	0.199	0.219	0.136
210% Taft (w/dampers)	0.268	0.336	0.193
262.5% Taft (w/dampers)	0.359	0.425	0.256
315% Taft (w/dampers)	0.425	0.498	0.288
420% Taft (w/dampers)	0.662	0.817	0.448

For earthquake tests through the 75-percent El Centro level, white noise tests (and visual observations) showed little further deterioration of the structure. Following the 100-percent (PGA ≈ 0.41g) and 150-percent El Centro tests, the fundamental frequency as determined by white noise tests decreased and widened in bandwidth; the average was approximately 1.38 Hz. During the 200-percent El Centro test, the structure failed. It was not catastrophic, but further testing was deemed to be unsafe. Failure occurred when the concrete cover over the reinforcement that extended from the third floor columns into the third floor slab spalled off, substantially softening the joint region. At nearly the same instant, the second and third floor spandrel beams failed in torsion at their interfaces with the columns.

A brief comparison of the 150-percent El Centro responses with and without dampers (Tables 1-4) shows a trend toward higher base shear forces and lower displacements with dampers installed. Maximum interstory drift, which occurred in the second story, with the dampers installed was 0.45 in. versus 1.21 in. without dampers. Maximum base shear with dampers installed was 34.6 kips, versus 17.0 kips without dampers. It is important not to draw incorrect conclusions from these observations, as the stiffness of the structure had decreased between the two 150-percent El Centro tests. The 200-percent El Centro test maxima are also listed in Tables 1-4; note that failure occurred in the unbraced structure at this acceleration amplitude.

Detailed analyses of the test data, including consideration of damper temperatures, displacements, and forces, are underway. In addition, analytical models are being used to interpret the observed responses. The preliminary results of these experiments indicate that damage can be controlled or limited in older reinforced concrete structures using supplemental dampers. Although the viscoelastic dampers performed well in the laboratory environment, design issues must be resolved before implementation in a full-scale seismic rehabilitation program. Key issues to be addressed include

damper sizing and placement, ambient temperature control, damper proof testing, and appropriate structural detailing. The technology looks promising, and tests of other damping devices are proposed.

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